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SOIL-SHEET PILE INTERACTION - PART I: A REVIEW OF THEORIES AND DESIGN METHODS

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ABSTRACT

This study reviews the theories and modelling methods of the interaction between soil and the embedded sheet pile structure covering the various responses and performance of both materials under various conditions.Interaction between soil and the embedded sheet pile structure is important considering the complex and indeterminate behaviour of both materials which results in non-linear structural deformation.The study affirms the importance of numerical computational tools in resolving the complex nature of interaction between soil and sheet-pile. Also, the study considers the contribution of various soil parameters in the interaction and the extent of response in terms of behavioure. deformation, stress distribution etc. It is foundthat the true behaviour and magnitude of deformation and stress distribution is generally dependent on the model and design technique chosen and all these in general contribute to the overall performance of both the soil and sheet pile wall.

Keyword: Interaction, Soil, Sheet-pile

1.0 INTRODUCTION

Indebt study and proper understanding of the complex nature of soil-sheet pile interaction is of great importance for improved design consideringthe associated non-linear contact behaviour. This non-linear and complex behaviour places much challenges in smooth-running of analysis of interaction between the soil and the embedded structure; in this case, sheet pile wall. Neglect to this interaction complexity has generally led to so much assumptions and near-exactness solution associated with conventional design of sheet pile walls.Some of these challenges include;

- Prediction of actual deformation.
- Over-conservativeness in design.
- Inaccuracy in stress and pressure distribution.

- Neglect to non-linearity behavior of soil.
- Inconsistency in answer resulting to quandary and uncertainty.

The need to proffer solution to these challenges in design of sheet pile and proper prediction of performance has been the area of interest among researchers (Ömer, 2012).

However, varieties of conventional design methods have been in use till date. These includes the fixed and free earth support method based on limiting equilibrium technique commonly found in standard codes such asEuroCode 7 (BS EN 1997), British Standard 8002 (BS 8002:1994) etc. with availability of several factors to carter for this assumption; but then, the real physical situations are not still well captured.

In this respect, an improved and higher computational technique is undeniably required to ensure a better design of sheet pile walls. The application of Finite Element Methods (FEM) in studying the interaction between soil and the sheet pile structure has been successfully employed in similar situations in previous studies and researches (Ebo, 2014). The use of FEM in soil-structure interaction studies have been in practice for years with great success starting from its first application by Clough (1969).

The FEM is generally accepted as a convenient and well established technique for solving complex engineering problems in civil, geotechnical, nuclear, mechanical engineering and other general sciences. Over time, tremendous advancements have been made in the past 25 years both in mathematical formulations and generalization of FEM used to solve field problems in various engineering analysis (Barkanov, 2001).

The ability of the computational FEM to provide smooth tools to run comprehensive and all inclusive analysis of sheet pile performance in soil in which it is embedded has given acceptable results of the interaction between both materials which includes stress distribution, sliding and overturning effect, deformation, ground movements, dewatering due to construction activities etc. This has made the FEM a widely accepted approach for studyingsoil-sheet pile interaction (Ebo, 2014).

Hence, adequate interaction performance of the interface between soil and sheet pile requires novel integration of material properties modelling for both contacting materials, compatibility model development, and robust interfacial interaction techniques (Olubanwo, 2013). This whole process entails the use of the proper material properties, on the right model development, and in the right way.

In this respect, the interaction between the two materials can be adequately accounted for, together with the unequivocal effectsof their non-linear behaviour. Further, accounting for the contributions of various soil properties in FEM generally gives rise to improved result compare to the theoretical methods / assumptions employed in the conventional design methods. The benefits of the improved method areenormous, ranging from reliability and cost-effectiveness which are the primary objectives of engineering designs. Overall, numerical analysis readily provides alternatives to overcome difficult and time consuming analytical problems especially in geotechnical designs (MacDonald 2011).

However, in spite of the enormous benefits accruing from application of this technique, proper application, indebt understanding of the FEM remains a challenge to design engineers, as wrong and inadequate understanding of the behaviour of the members may lead to deceitful results which would result to error in design (Ebo, 2014).

The overall aim of this research is to review various theories and modelling methods for studying soil-pile interaction, with focus on the structural behaviour in terms of stress and strain distribution, deformation, stability and structural integrity of sheet pile wall.

2.0 REVIEW OF THEORIES AND MODELLING METHODS

In solving soil-structure interaction, application of a more pragmatic approach is of importance other than just ensuring separation of the soil and sheet pile wall at loading. Application of numerical analysis by use of FEM is undeniably the most advanced tool for carrying out analysis of the interaction between soil and sheet pile taking into account both their material and geometric nonlinearities (Jahromi, 2009).

The applications of FEM have gained exponential rise in all engineering branches as well as in applied sciences; and in recent time, is undeniably the most used numerical method in both civil and geotechnical engineering and still gaining widespread popularity in other disciplines (Zienkiewicz and Taylor 1989).

The present day applications of FEM in geotechnics are reported to have originated from academic researchers from Northern America and Europe. Research in this line of subject hasbeen on the increase over the years by several scholars (Richard 2003). Various model types are presently in use with previous successful application in study of soil-structure interaction. These are discussed below.

2.1 Model types

Various kinds of soil – sheet pile models and techniques are presently in use, predominantly, the Mohr-Coulomb model and cap model with each of the models having advantages and associated limitations depending on the application (Mohammad et al., 2012).

A brief comparison of various soil models which have been in existence and used for various research works includes:

2.1.1 Mohr-Coulomb model:

The Mohr-Coulomb failure criterion takes into consideration the effects of stresses on the strength of soil when the equation (1) shown below is satisfied, i.e. the stress at any point in the material satisfies the equation;

$$\tau = c - \sigma tan\varphi \qquad (1)$$

Where, φ *and* $c$represent the angle of friction and cohesion of the soil respectively.

The Mohr-Coulomb criterion in principal stress term is generally given by:

$$\frac{1}{2}(\sigma_1 - \sigma_2) = -\frac{1}{2}(\sigma_1 + \sigma_2)sin\varphi + c(cos\varphi) \qquad (2)$$

However, in the principal stress space, the Mohr-Coulomb yield criterion exists in the form of a hexagonal cone with an invariant form as shown in equation (3) and Figure 1.

$$f_1 = \frac{I_1}{3}sin\varphi - \sqrt{\frac{J_2}{3}}sin\theta sin\varphi + \sqrt{J_2}cos\theta - c(cos\varphi) \qquad (3)$$

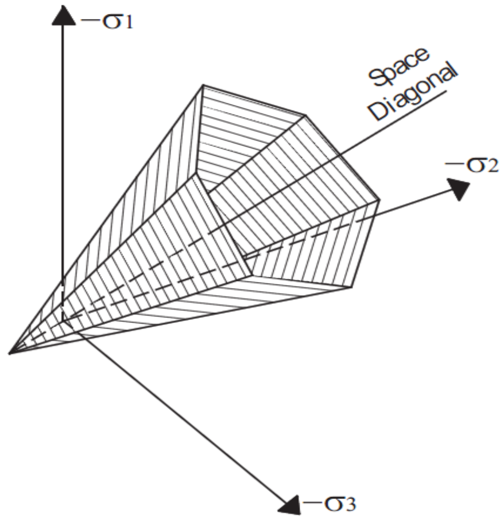

Figure 2.1: Principal stress space of Mohr-Coulomb failure criteria (Beekman et al 2000)

Figure 1 above shows the Mohr-Coulomb principle stress failure criteria with a representation of the various parameters arranged in space. Another function of interest is the plastic potential form which takes the same form as the yield function and can be presented in the Mohr-Coulomb criterion by the replacement of the angle of friction, θ by dilatancy angle, ψ in the yield function; this is given by equation 4:

$$g = \frac{I_1}{3}sin\psi - \sqrt{\frac{J_2}{3}}sin\theta sin\psi + \sqrt{J_2}cos\theta - c(cos\psi) \tag{4}$$

Where the dilatancy angle, ψ is used to model the dilatative plastic volumetric increments which comes into play in dense soils as exists in some of the soil strata. However, in real life situations, soil is incapable of sustaining much tensile stress, an act that can be stated by tension cut-off which can be represented with the functionsf_2, f_3, f_4 and the maximum tensile stress, T given in equation 5 (Goldscheider 1984):

$$f_2 = \sigma_3 - T, f_3 = \sigma_2 - T, f_4 = \sigma_1 - T \tag{5}$$

The Drucker-Prager model is a simplification of the Mohr-Coulomb model with a replacement of the hexagonal shape of the failure cone in Figure 1 with the simplified cone shown in Figure 2.2. The Drucker-Prager model shares same advantages and limitations with the Mohr-Coulomb model but is the most preferred due to its simplification and available in most recent FE packages such as ANSYS (Zhao et al 2008).

The Drucker-Prager plasticity is applicable to granular materials such as soils with its yield surface dependent on von Mises expression:

$$F = 3\beta\sigma_m + \frac{\sigma_{eqv}}{\sqrt{3}} - \sigma_y \tag{6}$$

Where σ_y = material yield, σ_m = von Mises stress, β = material constant.

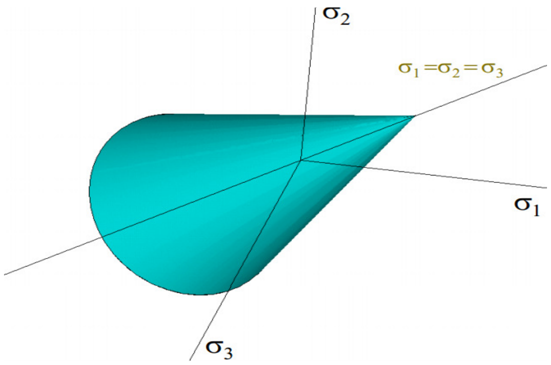

Figure 2.2: Drucker-Prager principle stress space (Leddin 2011)

2.1.2 Cap model

This is a form of soil constitutive model of the plasticity kind based on the concept of continuum mechanics and critical state expressed in terms of three-dimensional stress state. It is expressed in form of a dilative failure surface, f_1, contractive yield cap surface, f_2, each represented by the expressions:

$$f_1 = \sqrt{J_2} + \gamma e^{-\beta I_1} - \theta I_1 - \alpha = 0 \tag{7}$$

Where,β, α, γ and θrepresent the material parameters with the quantity $(\alpha - \gamma)$ used in material cohesive strength measurement.

$$f_2 = R^2 J_2 + (I_1 - C)^2 - R^2 b^2 = 0 \tag{8}$$

Where, R is shape factor, the other parameters are as defined in Figure 2.3.

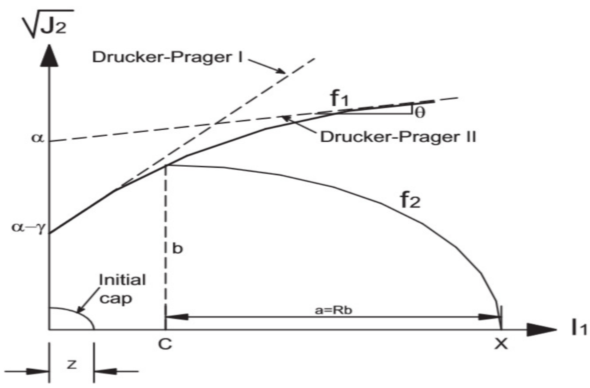

Figure 2.3: Cap model yield surface (Desai and Siriwardane, 1984)

3.0 CONVENTIONAL DESIGN THEORIES OF SHEET PILES

Several theories are in existence for determination of pressure distribution used for the design of sheet pile. The most prominent of these theories includes the Coulomb and Rankine theories with both made up of equations developed with fundamental assumptions that the retained soil is homogenous, drained and cohesionless (Barnes, 2010).

3.1 Coulomb Theory

The fundamental principle of the Coulomb's theory is the creation of a failure surface of soil in a flat orientation and produces a slippage plane with appropriateness of the assumption preferably applicable to active cases with significant errors encountered when used for the passive case. Other important hypothesis of the theory includes assumptions on the homogeneity of the soil, fulfilment of the Mohr-Coulomb failure criterion in the slippage plane, and the presence of soil angle of friction.
Another important requirement of the method includes adequate rotation/yielding of the wall for full engagement of all the shear strength, equilibrium of wedges at failure. A fundamental difference of the active and passive cases in this method includes formation of a high limit wedge to ensure engagement of the highest thrust while in the passive case, the wedge with provision of minimum/low thrust is formed.

Hence, with the above assumptions for soils in drained conditions, homogenous and cohesionless, linear earth pressure law for the active case is put into consideration using the relation;

$$\sigma_h = k_a \gamma Z \tag{9}$$

Where σ_h = horizontal pressure, γ = unit weight of soil, Z = height, k_a = coefficient of active pressure represented with the equation;

$$k_a = \frac{sin^2(\alpha + \emptyset)}{sin^2 \sin(\alpha - \delta)[\sqrt{\frac{\sin(\emptyset+\delta)\sin(\emptyset-\beta)}{\cos(\alpha-\delta)\cos(\alpha+\beta)}}} \tag{10}$$

Where α = angle of inclination of the wall with the horizontal.
$\emptyset$ = angle of friction of the soil.
β = angle of inclination of the ground behind the wall.
δ = angle of friction between the soil and wall (Barnes 2010).

A typical diagram of the Coulomb's wedge analysis showing the various angles as inclined is shown in Figure3.1.

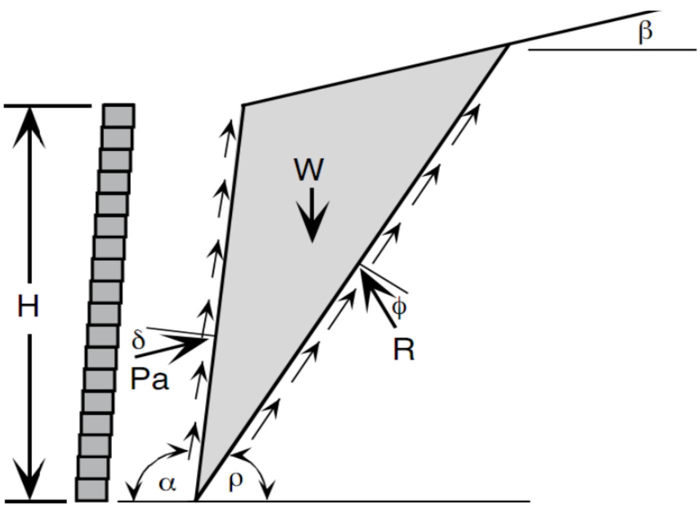

Figure 3.1: Coulomb wedge analysis showing the various angles (Barnes, 2010)

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It is evident from the Figure3.1 that the direction of the thrust is dependent on the angle of internal friction, δ with its magnitude calculated with consideration to equilibrium of forces at failure.

The Coulomb theory is known for its slight conservativeness and most appropriate for active cases and inappropriate for passive cases especially in situations of high δ and $\emptyset$ values, hence resulting to a critical condition for the sheet pile wall.

3.2 Rankine theory

The fundamental hypothesis of the Rankine theory is the condition of the soil still within Rankine limit state which is a stress state of plastic equilibrium with failure surfaces in two directions. Here the passive case involves more volume of the soil wedge than in the active case as shown in the Figure3.2.

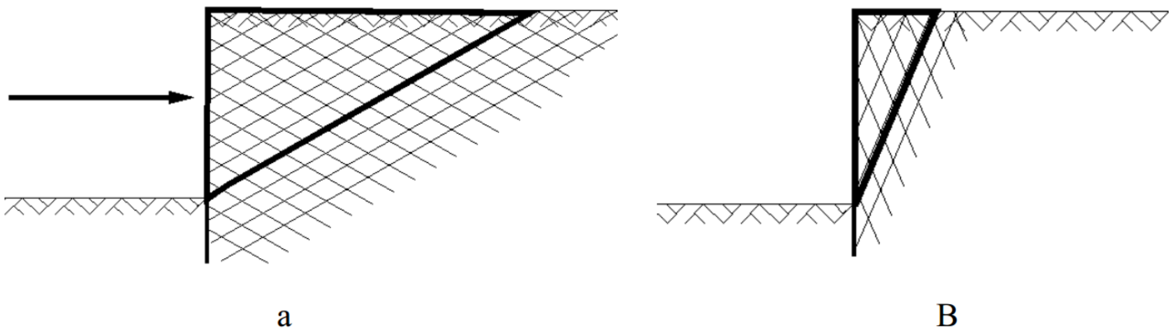

Figure 3.2: Rankine failure wedges (a) Passive limit state (b) Active limit state (Alejo 2013)

This takes place when the soil has attained the Mohr-Coulomb failure criterion represented with the equation:

$$\tau = c + \sigma tan\emptyset \tag{11}$$

Where τ = shear stress, c = cohesive stress, σ = normal stress, $\emptyset$ = angle of friction.

The failure criterion is hereby represented by Figure 3.3, showing the Mohr's circle and the Mohr-Coulomb failure envelope.

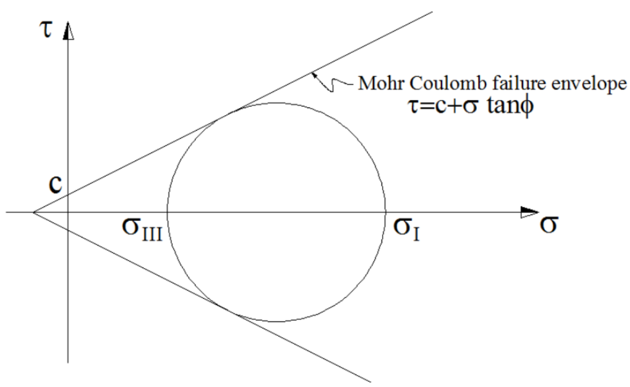

Figure 3.3: Mohr Coulomb failure envelope (Whitelow, 2001)

However, for the above criterion under the Rankine theory two frictional angles are required for the soil to fail at active and passive limit states respectively, this is shown in Figure3.4.

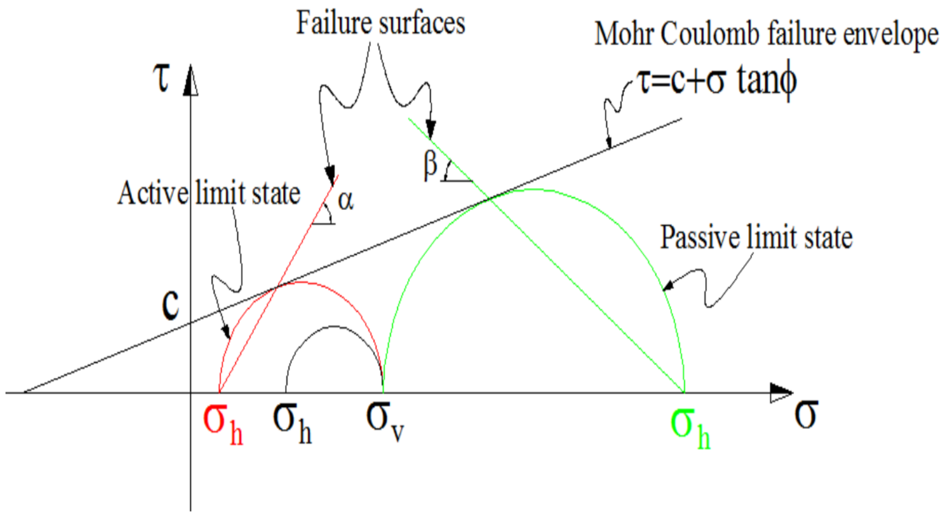

Figure 3.4: Mohr-Coulomb failure criterion (Alejo 2013)

In Figure 3.4, for the active state, the horizontal stress $\sigma_h$and the vertical stress σ_v represent the minor stress, $\sigma_3$and $\sigma_1$major stress, respectively. The active and passive stresses can be calculated with the following equations:

$$\sigma_h' = \sigma_v' K_a - 2c\sqrt{K_a} \text{ (Active case)} \qquad (12)$$

$$\sigma_h' = \sigma_v' K_p - 2c\sqrt{K_p} \text{ (Passive case)} \qquad (13)$$

Where $K_a = tan^2(\frac{\pi}{4} - \frac{\emptyset}{2})$

$$K_p = tan^2(\frac{\pi}{4} + \frac{\emptyset}{2})$$

In general, the Rankine theory is slightly conservative, adequate for active state and inadequate for passive state.

4.0 SHEET PILE WALLS AND DESIGN METHODS

Several design methods of sheet pile wall have been in existence either for cantilever or anchored sheet pile, with all establishing equilibrium of horizontal forces and moments so as to identify the failure state and applying appropriate safety factors. The design methods are described briefly below.

4.1 Cantilever Walls

For this case, the reaction force acts at the base of the wall and allows for stability where an increase in embedment depth results to increased reaction in the opposite direction, but is not a guarantee for stability as shown in Figure 4.1.

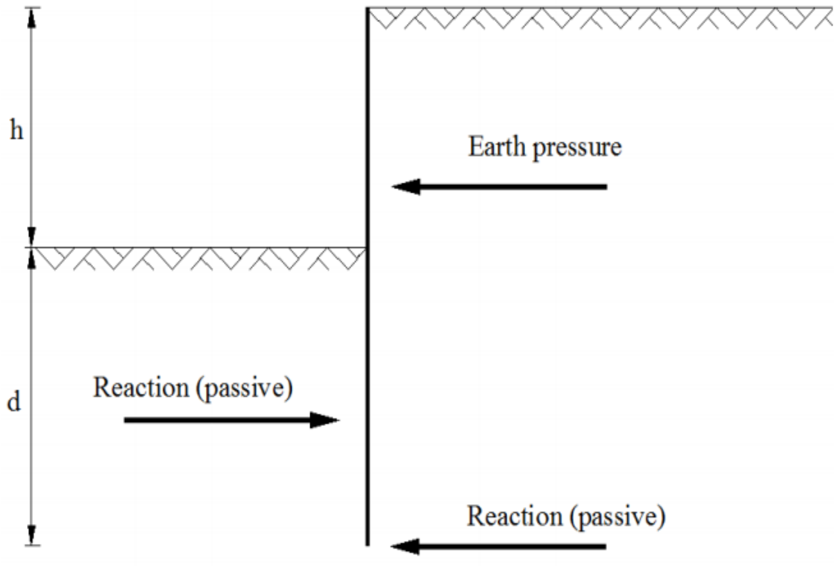

Figure 4.1: Net pressure for a cantilever wall (Alejo 2013)

It is evident from Figure4.1 that stability depends on the adequacy of embedment below the dredge line. Three main methods for design of cantilevered walls include; full, simplified and gradual method.

4.1.1 Full method

This involves the assumption that the active and passive limit states are reached at the back and in front of the wall respectively with a sudden jump in the earth pressure distribution as shown in Figure 4.2. The full method of design is popularly used in the UK.

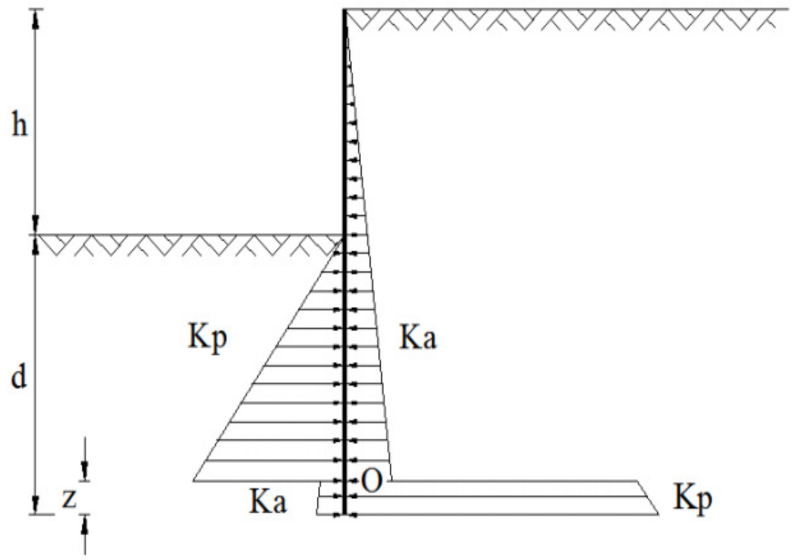

Figure 4.2: Earth pressure distribution for full method (Alejo 2013)

4.1.2 Simplified method

This involves a simplification of the full method due to its high complexity by replacing the earth pressure below the rotation point by a point load at point O as shown below:

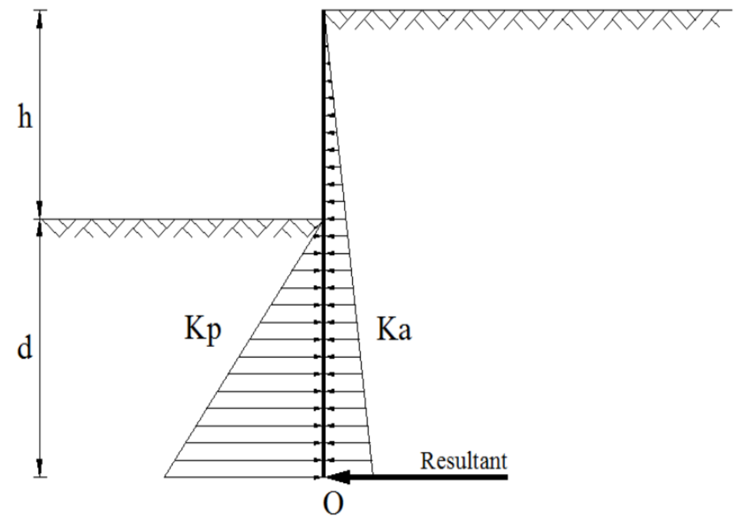

Figure 4.3: Simplified earth pressure (Alejo, 2013)

The simplification generally results in a reduction in embedded depth d, from the one obtained from the full method; this is corrected by a 20% to 40% increase of the d value i.e. using embedded depth ranging between 1.2d and 1.4d (Das, 2004).

4.1.3 Gradual method

Krey developed the method in 1932, later Bowles in 1988 reviewed the method which is based on considering a transition zone where there is a change in the direction of the net earth pressure i.e. at the point of rotation and assumed to be linear (Škrabl, 2006). This method is mostly used in U.S.A and is shown in Figure4.4.

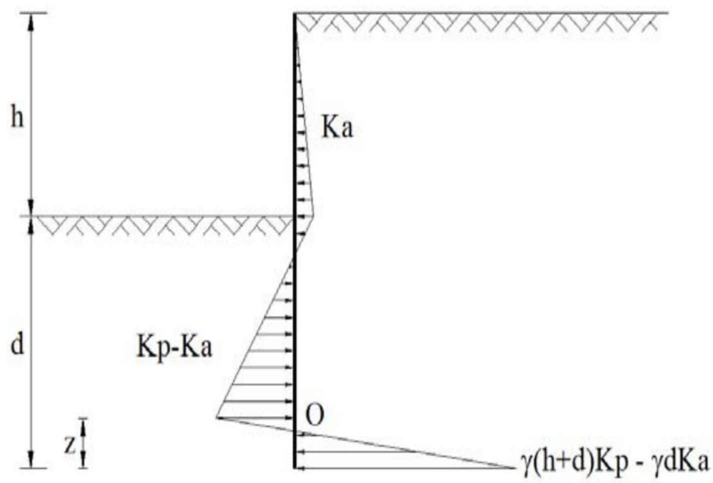

Figure 4.4: Net pressure for gradual method (Škrabl, 2006)

4.2 Anchored wall

Equilibrium can be actualized for the anchored wall type without putting the passive pressure at the bottom of the wall into consideration due tothe presence of anchor. Its main advantage over cantilever wall is possibility of embedded depth reduction which gives a more profitable structure; this is shown in Figure 4.5.

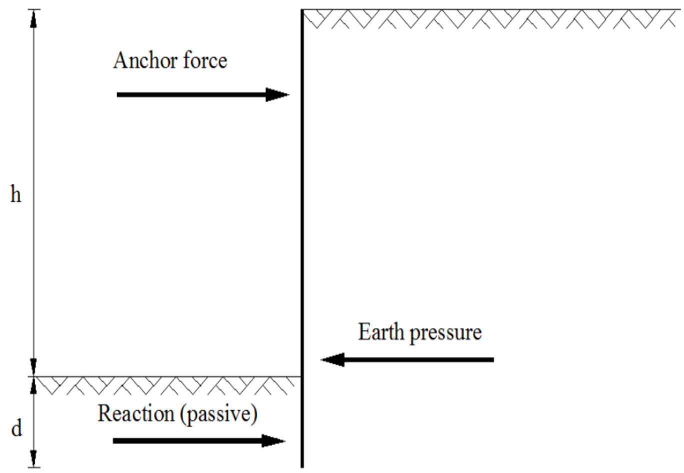

Figure 4.5: Net pressure distribution in anchored walls (Alejo, 2013)

The main methods used in design of anchored walls include; free and fixed earth support methods and are illustrated below (Das, 2004).

4.2.1 Free earth support method

This involves an assumed sufficiency of movements on the embedded region of the wall to mobilize both passive and active thrust at the front and behind the wall respectively with the bottom free to move. The embedment depth is obtained by using equilibrium satisfaction for both the active and passive pressures and anchor force (Alejo 2013); this is shown in Figure 4.6.

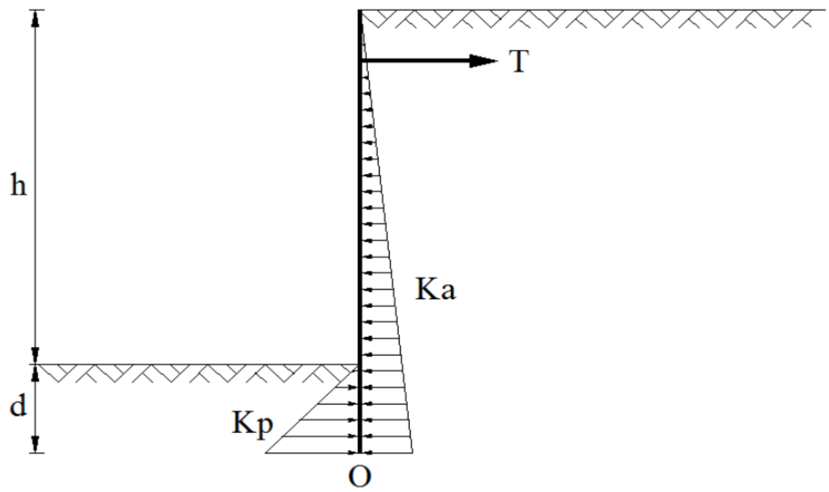

Figure 4.6: Pressure distribution for free earth support method (Das, 2004)

4.2.2 Fixed earth support method

For this method, the earth pressure distribution takes the form of a cantilever with the wall exhibiting the behaviour of a built-in beam exposed to bending moments and requires determining the point of zero net pressure (Das 2004). This is shown in Figure4.7.

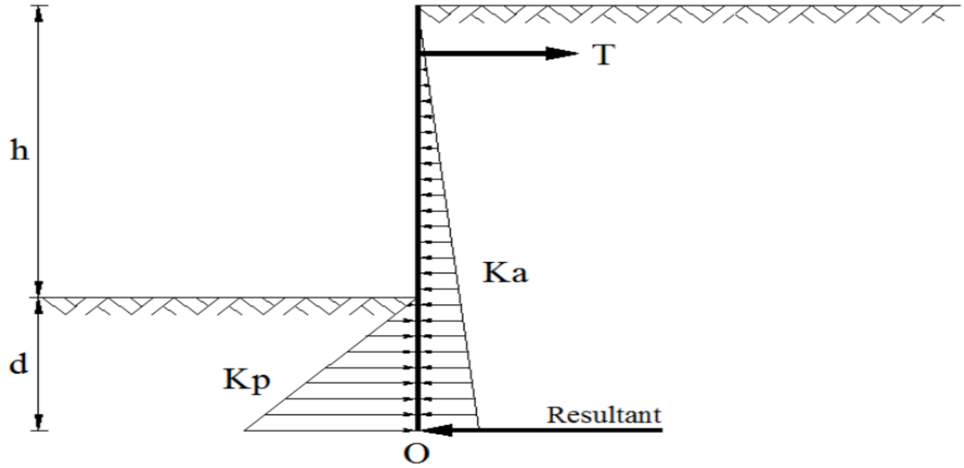

Figure 4.7: Earth pressure distribution for fixed earth support method(Alejo, 2013)

The above methods of sheet-pile design are currently in use and are applicable to the relevant standards such as the EuroCode 7 (BS EN 1997) and British Standard 8002 (BS 8002:1994) which use the serviceability and ultimate limit state procedure and consideration with factors for design of various site and material properties and conditions.However, no firm guideline exists for determining acceptable deflection of a sheet pile wall, but deflection ranging from 25mm to 127mm is generally acceptable (Techical supplement 14R 2007).

5.0 CONCLUSIONS

As have been reviewed on this paper, the following conclusions can be drawn;

- The true behaviour and magnitude of deformation, stress distribution etc. is dependent on model and design technique chosen and all these in general contribute to the overall performance of both the soil and sheet pile wall.
- No firm guideline exists currently in relevant design standards of sheet-piles for determining the acceptable deflection of a sheet pile wall.
- Much of the drawbacks associated with conventional analytical and design methods can be overcome using computational FEM of analysis / design by ensuring proper model creation, assigning sufficient material properties, choosing the right failure model, and ensuring that the solution captures the full geometric and material nonlinearities of the deformed configuration.

REFERENCES

1. Alejo, G. T. (2013) Numerical analysis of cantilever and anchored sheet pile walls at failure and comparison with classical methods. Unpublished dissertation. Barcelona: Escola de Camins.
2. Barkanov, E. (2001) Introduction to the Finite Element Method. Latvia: Riga Technical University
3. Barnes, G. (2010) Soil Mechanics – Principles and Practice.2nd edn. England: Palgrave Macmillan
4. Beekman, F., Badsi, M., and Wees, D. V. (2000) Faulting, Fracturing and In-Situ Stress Prediction in the Ahnet Basin, Algeria - A Finite Element Approach [online] available from <http://www.falw.vu/~beef/research/papers/ahnet/tp/ahnet.htm>[10 August 2014]
5. Clough, G. W. (1969) Finite element analysis of soil-structure interactions in U-frame locks. Unpublished PhD thesis. Berkeley: University of California

6. Das, B. M. (2004) Principles of foundation engineering. 5th edn. U.S.A: Brooks/Cole publishing company
7. Desai, C.S., and Siriwardane, H. J. (1984) Constitutive laws for engineering materials. Englewood Cliffs: Prentice-Hall Inc.
8. Ebo, E. K. (2014) Soil – sheet pile interaction. Unpublished dissertation. Coventry: Coventry University Coventry United Kingdom
9. Goldscheider, M. (1984) “True triaxial tests on dense sands”.Constitutive relations for soils. Rotterdam: Balkema
10. Jahromi, H. Z. Z. (2009) Partitioned Analysis of Nonlinear Soil-Structure Interaction. Unpublished PhD thesis. London: University of London and Imperial College London
11. Leddin, M. (2011) Structural mechanics simulation (CSM using finite element analysis). UK: ANSYS UK
12. MacDonald, B.J. (2011) Practical stress analysis with finite elements.2nd edn. Dublin: Glasnevin publishing
13. Mohammad, M. J., Syed H. G., Mohammad, R. J., Alistair B., Mir, K. Z. A. and Reza, M., (2012) Using Finite Element method for Pile-Soil Interface (through PLAXIS and ANSYS). Academic Journal of Civil Engineering and Construction Technology, University College Cork, Cork, Ireland, Mazandaran University, Sari Iran Jahad, Daneshgahi of Tarbiatmodares university, Tehran, Iran Vol. 3(10), pp. 256-272, November 2012
14. Ömer B. P.E., (2012) Lateral Earth Pressure Coefficients for Anchored Sheet Pile Walls - International Journal of Geomechanics ASCE, September/October 2012
15. Richard, I. W. (2003) The Application Of Finite Element Analysis to the Design of Embedded Retaining Walls. Unpublished PhD thesis. Surrey: University of Surrey
16. Škrabl, S. (2006) “Interactional approach of cantilever pile walls analysis”.Slovenica: ActaGeotechnicaSlovenica 2006/1
17. Techical supplement 14R (2007) Design and Use of Sheet Pile Walls in Stream Restoration and Stabilization Projects [online] available from
18. <http://directives.sc.egov.usda.gov/OpenNonWebContent.aspx?content=17828.wba> [10 August 2014]
19. Whitelow, R. (2001) Basic soil mechanics.4th edition. London: Pearson Educational Ltd
20. Zhao, M. H., Liu D., Zhang L., Jiang, C. (2008) 3D finite element analysis on pile-soil interaction of passive pile group. Institute of Geotechnical Engineering, Hunan University, Changsha 410082, China
21. Zienkiewicz, O. C. and Taylor, R. L. (1989) the Finite Element Method. 4th edition, Volume 1. Maidenhead: McGraw-Hill
22. Kavitha P. E., Dr. Beena K. S. and Dr. Narayanan K. P, “Analytical Study on Soil-Pile Interaction Effect In The Variation of Natural Frequency of A Single Pile” International Journal of Civil Engineering & Technology (IJCIET), Volume 5, Issue 12, 2014, pp. 226 - 229, ISSN Print: 0976 – 6308, ISSN Online: 0976 – 6316.
23. Nagendra Prasad.K, Manohara Reddy.R, Chandra.B and Harsha Vardhan Reddy.M, “Compression Behaviour of Natural Soils” International Journal of Civil Engineering & Technology (IJCIET), Volume 4, Issue 3, 2013, pp. 80 - 91, ISSN Print: 0976 – 6308, ISSN Online: 0976 – 6316.
24. Abeer Sabri Bshara, Er. Y. K .Bind and Prabhat Kumar Sinha, “Effect of Stone Dust on Geotechnical Properties of Poor Soil” International Journal of Civil Engineering & Technology (IJCIET), Volume 5, Issue 4, 2014, pp. 37 - 47, ISSN Print: 0976 – 6308, ISSN Online: 0976 – 6316.